Design Report

Team Name: “BRIDGE BUILDERS”

Team Registration Number: 0352

University name: “SYLHET ENGINEERING COLLEGE”

1. Project Information

Project Name: G + 07 (Eight) Storied Residential Building

Architectural design: En-Genius

Structural design: Bridge Builders

Description: It is an eight storied residential building, located in Chittagong city in Bangladesh. The structural design of that building has been prepared by team "bridge builders".

2. Design standard and codes, chosen structural system.

Design standard and codes:

- BNBC-2020
- Chattogram Mohanagar Imarat Nirman Bidhimala- 2008

Structural System: IMF-DUAL system: special reinforced concrete shear wall

3. Materials: all required engineering and design properties of materials, especially those of the reinforcement and concrete.

Materials	Engineering properties	Design properties
Concrete	Modulus of elasticity = 4700 $\sqrt{f'c}$	Compressive strength = 25 MPa
	Poisson's ratio = 0.2	Ultimate strain
	Unit weight = 24 kN/m^3	-
Steel	Modulus of elasticity = 4700 $\sqrt{f'c}$	yield strength = 420 MPa
	Poisson's ratio = 0.3	Expected yield strength
	Unit weight = 78.5 kN/m^3	Ultimate strain

4. Basic design loads

A. Dead load: floor finish, partition load, unit weight of plain and reinforced

- Floor finish = 1.2 kN/m^2 = 25 psf (Table 6.2.2, BNBC-2020)
- Partition load = 0.61 - 0.68 kN/m = 0.45 – 0.5 kip/ft (Table 6.2.1, BNBC-2020)
- Unit weight of plain concrete = 22.8 kN/m^3 (Table 6.2.1, BNBC-2020)
- Unit weight of reinforced concrete = 22.8 + (0.63 * 1% of main reinforcement) (Table 6.2.1, BNBC-2020)

B. Live load: on floors, kitchen and toilet and other areas. (Table 6.2.3, BNBC-2020)

- Typical floor (included kitchen) = 2 kN/m^2
- Balconies = 4.80 kN/m^2
- Roof = 4.80 kN/m^2
- Stair = 4.80 kN/m^2

C. Wind load: basic wind speed, exposure category, importance factor and other

Parameter	Value	Reference
Wind direction	X = 0^0, y = 90^0	-
Wind case	All case	Figure 6.2.8, BNBC-2020

Wind speed	80 m/s (179 mph)	Table 6.2.8, BNBC-2020
Surface roughness category	A	Sec 2.4.6, BNBC-2020
Exposure category	A	Sec 2.4.6.3, BNBC-2020
Topographic factor	1	Sec 2.4.7.2, BNBC-2020
Gust effect factor	0.85	Sec 2.4.8.1, BNBC-2020
Importance class	II	Table 6.1.1, BNBC-2020
Importance factor	1.00	Table 6.2.9, BNBC-2020
Wind directionality factor	0.85	Table 6.2.12, BNBC-2020
Wind loading method	Analytical procedure	Sec 2.4.3, BNBC-2020

Example wind load calculation (analytical procedure):

Step-1: velocity pressure

Directionality factor, k_d = 0.85 (table 6.2.11, bnbc-2020)
Topographic factor, k_{zt} = 1.00
Velocity pressure, v = 80 m/s
Importance factor, I = 1.00
Velocity pressure, $q_z = 0.000613 * k_z * k_{zt} * k_d * v^2 * I$
$= 0.000613 * 1 * 0.85 * (80)^2 * 1 * k_z$
$= (3.33 * k_z)$ kN/m^2
$= (M * k_z)$ kN/m^2

Exposure category = A
Case = 2
Height above ground level, z = 5 + 8*10 + 9.5 = 94.5 feet = 28.80 meter
Width of windward wall = 23.73 meter
Length of leeward wall = 8.86 meter

Step-2: wall pressure coefficients'

Surface	L/B	C_p
Windward wall	0.37	0.80
Leeward wall	0.37	-0.50
Side wall	0.37	-0.70

Step-3: Design wind-load calculations If z = 1.5 m,

K_z = 0.57
Q_z = 3.33 * 0.57 = 1.90 kN/m^2
$P_w = q_z * G * C_{p\_windward}$ = 1.90 * 0.85 * 0.80 = 1.29 kN/m^2
$P_l = q_z * G * C_{p\_leeward}$ = 3.23 * 0.85 * (-0.50) = - 1.37 kN/m^2
Net pressure, P_z = 1.29 + 1.42 kN/m^2 = 2.71 kN/m^2
Surface area = (0.76 + 1.525) * 23.73 = 54.24 m^2
Wind force, P_z= 54.24 * 2.66 = 144. 45 kN

Steo-4: Same calculation below the table according to height from above ground level-

Height above ground floor, Z (m)	Exposure coefficients, k_z	M	Velocity pressure, q_z (kN/m^2)	Wall pressure coefficients, c_p		Gust effect factor, g	External pressure		Net pressure, pp (kN/m^2)	Surface area, m^2	Wind force, P_z (kN)
				Windward wall	Leeward wall		Leeward	Windward			
1.5	0.57	3.33	1.90	0.80	-0.50	0.85	-1.37	1.29	2.66	54.24	144.45
4.6	0.57	3.33	1.90	0.80	-0.50	0.85	-1.37	1.29	2.66	72.31	192.59
7.6	0.66	3.33	2.20	0.80	-0.50	0.85	-1.37	1.49	2.87	72.31	207.34
10.7	0.73	3.33	2.43	0.80	-0.50	0.85	-1.37	1.65	3.03	72.31	218.81
13.7	0.78	3.33	2.60	0.80	-0.50	0.85	-1.37	1.77	3.14	72.31	226.99
16.8	0.83	3.33	2.76	0.80	-0.50	0.85	-1.37	1.88	3.25	72.31	235.18
19.8	0.87	3.33	2.90	0.80	-0.50	0.85	-1.37	1.97	3.34	72.31	241.73
22.9	0.91	3.33	3.03	0.80	-0.50	0.85	-1.37	2.06	3.43	72.31	248.28
25.9	0.94	3.33	3.13	0.80	-0.50	0.85	-1.37	2.13	3.50	70.51	246.88
28.8	0.97	3.33	3.23	0.80	-0.50	0.85	-1.37	2.20	3.57	34.35	122.60

Design wind load in X direction, Wind_X = 2084.86 kN = 468.696 kip

Ans.

D. Earthquake load: zone, zone coefficient, seismic design category, value of R

Parameter	Value	Reference
Seismic zone (Chattogram)	III	Figure 6.2.24, BNBC-2020
Seismic zone coefficient	0.28	Table 6.2.15, BNBC-2020
Building occupancy category	II	Table 6.1.1, BNBC-2020
Soil type	SD	SOIL TEST REPORT
Seismic design category	D	Table 6.2.18, BNBC-2020
Seismic force-resisting system	Dual system: IMF (special reinforced shear walls)	Table 6.2.19, BNBC-2020
Response reduction factor, r	6.5	Table 6.2.19, BNBC-2020
System overstrength factor, **Ω**	2.5	Table 6.2.19, BNBC-2020
Deflection amplification factor, c_d	5	Table 6.2.19, BNBC-2020
Spectral response acceleration parameter	S_s = 0.7, s_1 = 0.28	Table 6.C.1, BNBC-2020
Site coefficient	F_a = 1.35, F_y = 2.7	Table (6.C.1,6. C.2), BNBC2020
Occupancy importance factor	1.00	Table 6.2.17, BNBC-2020
Time period, t	0.631 second	Sec 2.5.7.2, BNBC-2020
Building height for time period calculation	30.33 meter (99.5 feet)	Sec 2.5.7.2, BNBC-2020
Time period coefficient	C_t = 0.0488, m =0.75	Table 6.2.20, BNBC-2020
Long period transition period	2 second	-

Example seismic base shear calculation.

Step-1: Time period of the building

Zone coefficient, z = 0.28
Importance coefficient, I = 1.00
Response reduction factor, r = 6.5
Viscous damping ratio, ζ = 5%
Soil type = SD
Height of the building = 30.33 meter (99.5 feet)
The building time period, T = 0.0488 * $(30.33)^{0.75}$ = 0.631 second
Site dependent soil factor and other deflecting elastic response spectrum (table 6.2.16, BNBC-2020)
S = 1.35; $T_B(s) = 0.20$; $T_C(s) = 0.80$

damping correction factor, $\eta = \sqrt{\frac{10}{5+\zeta}} = \sqrt{\frac{10}{5+5\%}} = 1 >= 0.55$

Step-2: design spectral acceleration

Normalized acceleration response spectrum, C_s = 2.5 * s * η (sec 2.5.4)
= 2.5 * 1.35 * 1
= 3.375 Design spectral acceleration, $S_a = \frac{2}{3} * \frac{Z*I}{R} * C_s = \frac{2*0.28*1*3.375}{3*6.5} = 0.096$

Which is less than = 0.67*βZIS = 0.67 * 0 .11 * 0.28 * 1 * 1.35 = 0.028 So, acceptable design spectral acceleration.

Step-3: seismic weight

Seismic weight (from analysis):
Deal load = 5068.64 kip
Live load = 650.734 kip
Live (roof) = 162. 066 kips
Live load (above 3 kN/m^2) = 313.712 kip
Total seismic weight, w = (5068.64 + 650.734 * 0.25 + 162.066 * 0.25 + 313.712 * 0.50) kip
= 5428.70 kip

Step-4: design base shear

Design base shear, v = S_a * w = 0.096 * 5428.70 = 521.15 kip = 2313.92 kN

Ans.

4

5. Load combinations

SI NO	Primary Load Combination	Load Combinations
1.	1.4D	
2.	1.2D + 1.6L + 0.5L_R	
3.	1.2D + 1.6L_R + (L OR 0.8 W)	1.2D + 1.6L_R + 1.0L
4.		1.2D + 1.6L_R + 0.8W(X)
5.		1.2D + 1.6L_R - 0.8W(X)
6.		1.2D + 1.6L_R + 0.8W(Y)
7.		1.2D + 1.6L_R - 0.8W(Y)
8.	1.2D + 1.6W + 1.0L + 0.5L_R	1.2D + 1.6W(X) +1.0L + 0.5L_R
9.		1.2D - 1.6W(X) + 1.0L + 0.5L_R
10.		1.2D + 1.6W(Y) +1.0L + 0.5L_R
11.		1.2D - 1.6W(Y) + 1.0L + 0.5L_R
12.	1.2D + 1.0E + 1.0L	1.2D + 1.0E(X) + 0.3E(Y) + EV +1.0L
13.		1.2D + 1.0E(X) - 0.3E(Y) + EV +1.0L
14.		1.2D - 1.0E(X) + 0.3E(Y) + EV +1.0L
15.		1.2D - 1.0E(X) - 0.3E(Y) + EV +1.0L
16.		1.2D + 0.3E(X) + 1.0E(Y) + EV +1.0L
17.		1.2D + 0.3E(X) - 1.0E(Y) + EV +1.0L
18.		1.2D - 0.3E(X) + 1.0E(Y) + EV +1.0L
19.		1.2D - 0.3E(X) - 1.0E(Y) + EV +1.0L
20.	0.9D + 1.6W	0.9D + 1.6W(X)
21.		0.9D - 1.6W(X)
22.		0.9D + 1.6W(Y)
23.		0.9D - 1.6W(Y)
24.	0.9D + 1.0E	0.9D + 1.0E(X) + 0.3E(Y) – EV
25.		0.9D + 1.0E(X) - 0.3E(Y) – EV
26.		0.9D - 1.0E(X) + 0.3E(Y) – EV
27.		0.9D - 1.0E(X) - 0.3E(Y) – EV
28.		0.9D + 0.3E(X) + 1.0E(Y) – EV
29.		0.9D + 0.3E(X) - 1.0E(Y) – EV
30.		0.9D - 0.3E(X) + 1.0E(Y) – EV
31.		0.9D - 0.3E(X) - 1.0E(Y) – EV

Notes:
W = Wind Load
E = Earthquake Load
EV = Earthquake Vertical Load
L_R = Roof of Live Loads

D = Dead Load L = Live Load

6. Deflection Data

A. **Maximum slab deflection (vertical) =** 16 mm (0.63 inch)

B. Maximum lateral sway due to wind and earthquake at floor levels

I. Maximum lateral sway (Displacement) due to wind in X direction

Story	Load Case/Combo	Direction	Maximum (in)
LIFT ROOF	1.0DL+0.5L+0.7W	X	1.509079
ROOF	1.0DL+0.5L+0.7W	X	1.390266
7TH FLOOR	1.0DL+0.5L+0.7W	X	1.228482
6TH FLOOR	1.0DL+0.5L+0.7W	X	1.054185
5TH FLOOR	1.0DL+0.5L+0.7W	X	0.866706
4TH FLOOR	1.0DL+0.5L+0.7W	X	0.669501
3RD FLOOR	1.0DL+0.5L+0.7W	X	0.470115
2ND FLOOR	1.0DL+0.5L+0.7W	X	0.280864
1ST FLOOR	1.0DL+0.5L+0.7W	X	0.12031

II. Maximum lateral sway (Displacement) due to wind in Y direction

Story	Load Case/Combo	Direction	Maximum (in)
LIFT ROOF	1.0DL+0.5L+0.7W	Y	0.387633
ROOF	1.0DL+0.5L+0.7W	Y	0.363098
7TH FLOOR	1.0DL+0.5L+0.7W	Y	0.336548
6TH FLOOR	1.0DL+0.5L+0.7W	Y	0.301849
5TH FLOOR	1.0DL+0.5L+0.7W	Y	0.259101
4TH FLOOR	1.0DL+0.5L+0.7W	Y	0.211967
3RD FLOOR	1.0DL+0.5L+0.7W	Y	0.161708
2ND FLOOR	1.0DL+0.5L+0.7W	Y	0.107317
1ST FLOOR	1.0DL+0.5L+0.7W	Y	0.052877

III. Maximum lateral sway (Displacement) due to earthquake in X direction

Story	Load Case/Combo	Direction	Maximum (in)
LIFT ROOF	EQ-X 1	X	2.534323
ROOF	EQ-X 1	X	2.437084
7TH FLOOR	EQ-X 1	X	2.149292
6TH FLOOR	EQ-X 1	X	1.838488
5TH FLOOR	EQ-X 1	X	1.503227
4TH FLOOR	EQ-X 1	X	1.151843
3RD FLOOR	EQ-X 1	X	0.800105
2ND FLOOR	EQ-X 1	X	0.471451
1ST FLOOR	EQ-X 1	X	0.198441

IV. Maximum lateral sway (Displacement) due to earthquake in Y direction

Story	Load Case/Combo	Direction	Maximum (in)
LIFT ROOF	EQ-Y 1	Y	2.743761
ROOF	EQ-Y 1	Y	2.543454
7TH FLOOR	EQ-Y 1	Y	2.316068
6TH FLOOR	EQ-Y 1	Y	2.033492
5TH FLOOR	EQ-Y 1	Y	1.703848
4TH FLOOR	EQ-Y 1	Y	1.343354

3RD FLOOR	EQ-Y 1	Y	0.970388
2ND FLOOR	EQ-Y 1	Y	0.606912
1ST FLOOR	EQ-Y 1	Y	0.279309

C. Seismic floor + torsional effect check

I. Torsional effect due to earthquake in X direction

Story	Load Case/Combo	Item	Max Drift	Avg Drift	Ratio
LIFT ROOF	EQ-X 1	Diaph D1 X	0.002162	0.00213	1.015
ROOF	EQ-X 1	Diaph D1 X	0.002398	0.002359	1.017
7TH FLOOR	EQ-X 1	Diaph D1 X	0.002593	0.002561	1.012
6TH FLOOR	EQ-X 1	Diaph D1 X	0.002799	0.002743	1.021
5TH FLOOR	EQ-X 1	Diaph D1 X	0.002937	0.002843	1.033
4TH FLOOR	EQ-X 1	Diaph D1 X	0.002945	0.002808	1.049
3RD FLOOR	EQ-X 1	Diaph D1 X	0.002756	0.002581	1.068
2ND FLOOR	EQ-X 1	Diaph D1 X	0.002295	0.002093	1.096
1ST FLOOR	EQ-X 1	Diaph D1 X	0.001428	0.001231	1.16

II. Torsional effect due to earthquake in X dir. + eccentricity

Story	Load Case/Combo	Item	Max Drift	Avg Drift	Ratio
LIFT ROOF	EQ-X 2	Diaph D1 X	0.002388	0.002281	1.047
ROOF	EQ-X 2	Diaph D1 X	0.002595	0.00234	1.109
7TH FLOOR	EQ-X 2	Diaph D1 X	0.002836	0.002528	1.122
6TH FLOOR	EQ-X 2	Diaph D1 X	0.003019	0.002708	1.115
5TH FLOOR	EQ-X 2	Diaph D1 X	0.003095	0.002806	1.103
4TH FLOOR	EQ-X 2	Diaph D1 X	0.003013	0.002769	1.088
3RD FLOOR	EQ-X 2	Diaph D1 X	0.00272	0.002543	1.07
2ND FLOOR	EQ-X 2	Diaph D1 X	0.002142	0.002058	1.041
1ST FLOOR	EQ-X 2	Diaph D1 X	0.001246	0.001201	1.038

III. Torsional effect due to earthquake in X dir. - eccentricity

Story	Load Case/Combo	Item	Max Drift	Avg Drift	Ratio
LIFT ROOF	EQ-X 3	Diaph D1 X	0.002021	0.001977	1.022
ROOF	EQ-X 3	Diaph D1 X	0.002712	0.002377	1.141
7TH FLOOR	EQ-X 3	Diaph D1 X	0.002966	0.002595	1.143
6TH FLOOR	EQ-X 3	Diaph D1 X	0.003202	0.002777	1.153
5TH FLOOR	EQ-X 3	Diaph D1 X	0.003358	0.00288	1.166
4TH FLOOR	EQ-X 3	Diaph D1 X	0.003364	0.002846	1.182
3RD FLOOR	EQ-X 3	Diaph D1 X	0.003146	0.002619	1.201
2ND FLOOR	EQ-X 3	Diaph D1 X	0.002617	0.002129	1.229
1ST FLOOR	EQ-X 3	Diaph D1 X	0.00161	0.001261	1.277

IV. Torsional effect due to earthquake in Y direction

Story	Load Case/Combo	Item	Max Drift	Avg Drift	Ratio

LIFT ROOF	EQ-Y 1	Diaph D1 Y	0.001899	0.001828	1.039
ROOF	EQ-Y 1	Diaph D1 Y	0.001895	0.001849	1.025
7TH FLOOR	EQ-Y 1	Diaph D1 Y	0.002355	0.002283	1.032
6TH FLOOR	EQ-Y 1	Diaph D1 Y	0.002747	0.002642	1.04
5TH FLOOR	EQ-Y 1	Diaph D1 Y	0.003004	0.002867	1.048
4TH FLOOR	EQ-Y 1	Diaph D1 Y	0.003108	0.002947	1.055
3RD FLOOR	EQ-Y 1	Diaph D1 Y	0.003029	0.002858	1.06
2ND FLOOR	EQ-Y 1	Diaph D1 Y	0.00273	0.002575	1.06
1ST FLOOR	EQ-Y 1	Diaph D1 Y	0.002015	0.001845	1.092

V. Torsional effect due to earthquake in Y dir. + eccentricity

Story	Load Case/Combo	Item	Max Drift	Avg Drift	Ratio
LIFT ROOF	EQ-Y 2	Diaph D1 Y	0.001865	0.001814	1.028
ROOF	EQ-Y 2	Diaph D1 Y	0.001945	0.001858	1.047
7TH FLOOR	EQ-Y 2	Diaph D1 Y	0.002412	0.002295	1.051
6TH FLOOR	EQ-Y 2	Diaph D1 Y	0.002811	0.002658	1.058
5TH FLOOR	EQ-Y 2	Diaph D1 Y	0.003073	0.002886	1.065
4TH FLOOR	EQ-Y 2	Diaph D1 Y	0.003179	0.002968	1.071
3RD FLOOR	EQ-Y 2	Diaph D1 Y	0.003096	0.00288	1.075
2ND FLOOR	EQ-Y 2	Diaph D1 Y	0.002786	0.002594	1.074
1ST FLOOR	EQ-Y 2	Diaph D1 Y	0.002033	0.001854	1.096

VI. Torsional effect due to earthquake in Y dir. - eccentricity

Story	Load Case/Combo	Item	Max Drift	Avg Drift	Ratio
LIFT ROOF	EQ-Y 3	Diaph D1 Y	0.001934	0.001825	1.059
ROOF	EQ-Y 3	Diaph D1 Y	0.001845	0.00184	1.002
7TH FLOOR	EQ-Y 3	Diaph D1 Y	0.002297	0.00227	1.012
6TH FLOOR	EQ-Y 3	Diaph D1 Y	0.002683	0.002626	1.022
5TH FLOOR	EQ-Y 3	Diaph D1 Y	0.002935	0.002849	1.03
4TH FLOOR	EQ-Y 3	Diaph D1 Y	0.003037	0.002926	1.038
3RD FLOOR	EQ-Y 3	Diaph D1 Y	0.002962	0.002837	1.044
2ND FLOOR	EQ-Y 3	Diaph D1 Y	0.002674	0.002556	1.046
1ST FLOOR	EQ-Y 3	Diaph D1 Y	0.002012	0.001843	1.091

VII. Seismic floor Due to earthquake in X direction

Story	H(m)	$C_d = 5$		I = 1		Δa = 0.02
		Elastic Displacement(mm)	**Amplified Displacement**	**Story Drift**	**Allowable**	**Check**
		δ	**Δm**	**Δi**	**Δa**	
WATER TANK	1.5	65	325	3.14	30	Safe
LIFT ROOF	2.9	64.372	321.86	12.35	58	Safe
ROOF	3	61.902	309.51	36.55	60	Safe

7TH FLOOR	3	54.592	272.96	39.47	60	Safe
6TH FLOOR	3	46.698	233.49	42.58	60	Safe
5TH FLOOR	3	38.182	190.91	44.625	60	Safe
4TH FLOOR	3	29.257	146.285	44.67	60	Safe
3RD FLOOR	3	20.323	101.615	41.74	60	Safe
2ND FLOOR	3	11.975	59.875	34.675	60	Safe
1ST FLOOR	3	5.04	25.2	5.2	60	Safe
1B GF	1.5	4	20	20	30	Safe
1A BASE	0	0	0	0	0	Safe

VIII. Seismic floor due to earthquake in X dir. + eccentricity

Story	**H(m)**	$C_d = 5$		I = 1		$\Delta a = 0.02$
		Elastic Displacement(mm)	**Amplified Displacement**	**Story Drift**	**Allowable**	**Check**
		δ	**Δm**	**Δi**	**Δa**	
WATER TANK	1.5	72	360	9.83	30	Safe
LIFT ROOF	2.9	70.034	350.17	34.57	58	Safe
ROOF	3	63.12	315.6	39.295	60	Safe
7TH FLOOR	3	55.261	276.305	42.92	60	Safe
6TH FLOOR	3	46.677	233.385	45.7	60	Safe
5TH FLOOR	3	37.537	187.685	46.895	60	Safe
4TH FLOOR	3	28.158	140.79	45.685	60	Safe
3RD FLOOR	3	19.021	95.105	41.28	60	Safe
2ND FLOOR	3	10.765	53.825	31.83	60	Safe
1ST FLOOR	3	4.399	21.995	6.995	60	Safe
1B GF	1.5	3	15	15	30	Safe
1A BASE	0	0	0	0	0	Safe

IX. Seismic floor due to earthquake in X dir. - eccentricity

Story	**H(m)**	$C_d = 5$		I = 1		$\Delta a = 0.02$
		Elastic Displacement(mm)	**Amplified Displacement**	**Story Drift**	**Allowable**	**Check**
		δ	**Δm**	**Δi**	**Δa**	
WATER TANK	1.5	60	300	5.235	30	Safe
LIFT ROOF	2.9	58.953	294.765	-4.865	58	Safe
ROOF	3	59.926	299.63	36.4	60	Safe
7TH FLOOR	3	52.646	263.23	39.45	60	Safe
6TH FLOOR	3	44.756	223.78	42.215	60	Safe
5TH FLOOR	3	36.313	181.565	43.76	60	Safe
4TH FLOOR	3	27.561	137.805	43.235	60	Safe

3RD FLOOR	3	18.914	94.57	39.775	60	Safe
2ND FLOOR	3	10.959	54.795	32.315	60	Safe
1ST FLOOR	3	4.496	22.48	2.48	60	Safe
1B GF	1.5	4	20	20	30	Safe
1A BASE	0	0	0	0	0	Safe

X. Seismic floor due to earthquake in Y dir.

Story	H(m)	$C_d = 5$		I = 1		Δa = 0.02
		Elastic Displacement(mm)	**Amplified Displacement**	**Story Drift**	**Allowable**	**Check**
		δ	**Δm**	**Δi**	**Δa**	
WATER TANK	1.5	71	355	4.02	30	Safe
LIFT ROOF	2.9	70.196	350.98	25.93	58	Safe
ROOF	3	65.01	325.05	29.235	60	Safe
7TH FLOOR	3	59.163	295.815	36.23	60	Safe
6TH FLOOR	3	51.917	259.585	42.18	60	Safe
5TH FLOOR	3	43.481	217.405	46.065	60	Safe
4TH FLOOR	3	34.268	171.34	47.61	60	Safe
3RD FLOOR	3	24.746	123.73	46.365	60	Safe

2ND FLOOR	3	15.473	77.365	41.765	60	Safe
1ST FLOOR	3	7.12	35.6	10.6	60	Safe
1B GF	1.5	5	25	25	30	Safe
1A BASE	0	0	0	0	0	Safe

XI. Seismic floor due to earthquake in Y dir. + eccentricity

Story	H(m)	$C_d = 5$		I = 1		Δa = 0.02
		Elastic Displacement(mm)	**Amplified Displacement**	**Story Drift**	**Allowable**	**Check**
		δ	**Δm**	**Δi**	**Δa**	
WATER TANK	1.5	73	365	6.215	30	Safe
LIFT ROOF	2.9	71.757	358.785	26.505	58	Safe
ROOF	3	66.456	332.28	30	60	Safe
7TH FLOOR	3	60.456	302.28	37.1	60	Safe
6TH FLOOR	3	53.036	265.18	43.155	60	Safe
5TH FLOOR	3	44.405	222.025	47.115	60	Safe
4TH FLOOR	3	34.982	174.91	48.685	60	Safe
3RD FLOOR	3	25.245	126.225	47.385	60	Safe
2ND FLOOR	3	15.768	78.84	42.625	60	Safe
1ST FLOOR	3	7.243	36.215	9.495	60	Safe
1B GF	1.5	5.344	26.72	26.72	30	Safe
1A BASE	0	0	0	0	0	Safe

XII. Seismic floor due to earthquake in Y dir. - eccentricity

Story	H(m)	$C_d = 5$		I = 1		Δa = 0.02
		Elastic Displacement(mm)	**Amplified Displacement**	**Story Drift**	**Allowable**	**Check**
		δ	**Δm**	**Δi**	**Δa**	
WATER TANK	1.5	70	350	6.83	30	Safe
LIFT ROOF	2.9	68.634	343.17	25.345	58	Safe
ROOF	3	63.565	317.825	28.475	60	Safe
7TH FLOOR	3	57.87	289.35	35.355	60	Safe
6TH FLOOR	3	50.799	253.995	41.21	60	Safe
5TH FLOOR	3	42.557	212.785	45.015	60	Safe
4TH FLOOR	3	33.554	167.77	46.535	60	Safe
3RD FLOOR	3	24.247	121.235	45.345	60	Safe
2ND FLOOR	3	15.178	75.89	40.91	60	Safe
1ST FLOOR	3	6.996	34.98	9.98	60	Safe
1B GF	1.5	5	25	25	30	Safe
1A BASE	0	0	0	0	0	Safe

7. Sum of the column reactions at base level and average load per unit floor area for all Basic load cases.

Sum of the column reactions at base level = 6195.156 Kip Area of the floor = 2513.69 sq. Ft.

Average load per unit floor area = $\frac{6195.156}{2513.69}$ = 2.4645 Kip / sq. Ft. (per unit floor area)

8. Base shear due to wind load as found from analysis

Load Case/Combo	FX	FY
	kip	kip
WIND-X 1	-448.534	0
WIND-X 2	0	-172.959
WIND-X 3	-336.401	0
WIND-X 4	-336.401	0
WIND-X 5	0	-129.719
WIND-X 6	0	-129.719
WIND-X 7	-336.401	129.719
WIND-X 8	-336.401	-129.719
WIND-X 9	-252.525	97.376
WIND-X 10	-252.525	97.376
WIND-X 11	-252.525	-97.376
WIND-X 12	-252.525	-97.376
WIND-Y 1	0	-172.959
WIND-Y 2	448.534	0
WIND-Y 3	0	-129.719
WIND-Y 4	0	-129.719
WIND-Y 5	336.401	0
WIND-Y 6	336.401	0
WIND-Y 7	-336.401	-129.719
WIND-Y 8	336.401	-129.719
WIND-Y 9	-252.525	-97.376
WIND-Y 10	-252.525	-97.376
WIND-Y 11	252.525	-97.376
WIND-Y 12	252.525	-97.376

9. Base shear due to earthquake load as found from analysis

Load Case/Combo	FX	FY
	kip	kip
EQ-X 1	-523.224	0
EQ-X 2	-523.224	0
EQ-X 3	-523.224	0
EQ-Y 1	0	-521.567
EQ-Y 2	0	-521.567
EQ-Y 3	0	-521.567

10. Typical foundation design calculation

Direction of load application: Gravity
Load combination: 1.2 DL + 1.6 LL + 0.5 L_r
Assume,
Length of pile cap = 100 in
Width of pile cap = 100 in
Pile diameter, D = 20 in

Step-1: Calculate pile number
Pile spacing, 3D = 60 in
Unfactored load, DL + LL = 402.50 kips (From Etabs Analysis)
Factored load, 1.2 DL + 1.2 LL + 0.5 L_r = 435 kips (From Etabs Analysis)
Ultimate capacity of pile, Q_u = 318.03 Kips (From soil report)
Safety factor = 2.5
Allowable capacity of Pile, q_a = 127.21 kips (From soil report)
Compressive strength of concrete, $f'c$ = 3500 psi Yield strength of steel, f_y = 60000 psi
Number of Pile = $\frac{402.50}{127.21} = 3.1467 \approx 4$ NOS
Applied factored force/pile, $P_{net} = \frac{435}{4}$ = 108.75 kip
Assume,
Effective depth, d = 18 in

Step-2: punching shear check
Allowable punching shear strength = $4\,\varphi\,\sqrt{f'c}$ = 4* 0.75 * $\sqrt{3500}$ = 177.48 psi
Punching shear force, $v_u = p_{net}$ * 4 = 108.75 * 4 = 435 kips
Actual punching shear stress = $\frac{Vu}{\text{Punching Perimeter}*d} = \frac{435*1000}{2*(38+33)*18}$ = 170.187793 psi Which is less than allowable punching shear stress.
So, adequate punching shear.

Step-3: One way/beam shear check
Allowable critical shear strength = $2\,\varphi\,\sqrt{f'c}$ = 4* 0.75 * $\sqrt{3500}$ = 88. 74 psi
Actual Critical shear force, V_f = 72.50 % of pile
Actual critical shear force, $V_f = 0.725 * 1000 * 108.75 * 2$ = 157687.5 lb
Actual critical shear stress, $\frac{725*1000*108.75*2}{100*18}$ = 87.60 psi
Which is less than allowable critical strength.

So, adequate one way or beam shear.

Step-4: Moment calculation Moment in short section, $m_{short} = \frac{2*pnet*l}{Length} = \frac{2*108.75*20}{8.33} = 522.20$ kip- in/ft

Moment in long section, $m_{long} = \frac{2*pnet*l}{Length} = \frac{2*108.75*22.5}{8.33} = 587.25$ kip-in/ft

Step-5: Effective depth check

Balanced reinforcement ratio, $\rho b = \alpha * \beta * \frac{f'c}{fy} * \frac{\epsilon uc}{\epsilon uc + \epsilon ut} = 0.85 * 0.85 * \frac{3500}{60000} * \frac{0.003}{0.003+.005} = 0.015804$

Maximum reinforcement ratio, ρmax = 0.75* ρb = 0.75 * 0.015804 = 0.0118535 Maximum moment, m = $\varphi * \rho * fy * b * d^2 * (1 - 0.59 * \rho * \frac{fy}{f'c})$

$$d^2 = \frac{587.25}{60 * 0.9 * 0.0118535 * 12 * (1 - 0.59 * 0.0118535 * \frac{60000}{3500})}$$

d = 9.32 in

Which is less than effective depth.

So, adequate effective depth.

Step-6: Longitudinal reinforcement

$A_s = \frac{587.25}{0.9*60*(18-(\frac{1.04}{2}))} = 0.622$ in^2/ft

$A_{s, min} = \frac{200}{60000} * 12 * 18 = 0.72$ in^2/ft (Controlling)

$A_{s, min} = \frac{3\sqrt{3500}}{60000} * 12 * 18 = 0.64$ in^2/ft

Using φ 20 mm bar

Spacing = $\frac{0.48*12}{0.72}$ = 8 in c/c

No. 6 (No. 20) bars are selected and placed 200 mm (8 in) c/c.

Step-8: Transverse reinforcement

$A_s = \frac{522.20}{0.9*60*(18-(\frac{0.92}{2}))} = 0.551$ in^2/ft

$A_{s, min} = \frac{200}{60000} * 12 * 18 = 0.72$ in^2/ft (Controlling)

$A_{s, min} = \frac{3\sqrt{3500}}{60000} * 12 * 18 = 0.64$ in^2/ft

Using φ 20 mm bar

Spacing = $\frac{0.48*12}{0.72}$ = 8 in c/c No. 6 (No. 20) bars are selected and placed 200 mm (8 in) c/c.

11. Typical column reinforcement design calculation

Direction of load application: Gravity
Load combination: 1.2 DL + 1.6 LL + 0.5 L_r

Assume,
Column width = 15 in
Column depth = 15 in

Step-1: Load calculations Unit
weight of concrete = 150 lb/ft^3
Number of storeys = 8
Compressive strength of concrete, $f'c$ = 3500 psi
Yield strength of steel, f_y = 60000 psi
Total length of Beam, L = 24.9 ft
Storey height = 10 ft
Length of partition wall, L = 24.9 ft
Supported Area = 193.7 sq. Ft. Slab
thickness = 5 in
Beam (X direction) = 10 in X 15 in
Beam (Y direction) = 10 in X 18 in
For one storey:
Dead Load:
Floor Finish, FF = 25 Psf
Uniform distributed load of beam (Y-direction) = 135.42 lb/ft
Uniform distributed load of beam (X-direction) = 104.17 lb/ft
Self-weight of beam = 135.42 * 12.3 + 104.17 * 12.6 = 2978.208 lb = 2.98 Kip
Self-weight of Partition on beam = 0.45 * 24.9 = 11.205 Kip
Self-weight of slab = (5 * 193.7 * 150) / (12*1000) = 12.11 Kip
Weight of floor finish = (25 * 192.7) / 1000 = 4.82 Kip
Total dead load for one storey = 12.11 + 4.82 + 11.205 + 2.98 = 31.115 Kip **Live load:**
Live load, LL = 42 Psf
Typical floor = (42 * 193.7) /1000 = 8.12 Kip

Ultimate load for one storey = 1.2 * 31.115 + 1.6 * 8.12 = 50.33 Kip

Roof:
Dead load:
Floor Finish, FF = 30 Psf
Live load, L_r = 100 Psf
Self-weight of slab = (5 * 193.7 * 150) / (12*1000) = 12.11 Kip
Weight of floor finish = (30 * 192.7) / 1000 = 5.78 Kip
Self-weight of beam = 135.42 * 12.3 + 104.17 * 12.6 = 2978.208 lb = 2.98 Kip
Total dead load for roof slab = 12.11 + 5.78 + 2.98 = 20.87 Kip **Live load**:
Roof slab = (100*193.7) / 1000 = 19.37 Kip
Ultimate load for roof slab = 1.2 * 20.87 + 0.5 * 19.37 = 34.78 Kip

Self-weight of column = (15 * 15 * 85 * 150) / (144*1000) = 19.92 Kip
Ultimate weight of column = 1.2 * 19.92 = 23.91 Kips

Total ultimate load for 8 storeys = 50.33 * 7 + 34. 78 + 23.91 + 2.98*1.2 = 414.58 Kip

Step-2: Gross area of column section
Ultimate load, P_u = 414.58 Kip
Strength reduction factor, φ = 0.65
Effective led factor, K = 0.80
Assume,
Reinforcement ratio = 2.5 %
Ultimate load, $P_u = \varphi * K * A_g [0.85 * f'c + p_g *(f_y - 0.85 * f'c]$

⇨ $\frac{414.58}{80*0.65} = A_g (0.85 * 3.5 + 0.025* (60 - 0.85 * 3.5)$
0.

⇨ $A_g = \frac{797.26}{4.40}$ in^2

⇨ A_g = 181.19 in^2

Step-3: Column reinforcement
For, width = 15 in = 381 mm
Depth = 12.1 ≈ 15 in = 381 mm
Gross area of Column, A_g = 225 in^2 = 145461 mm^2
Steel area, A_s = 0.025 * 225 = 5.625 in^2

Choose eight bars, 25 mm in diameter (160 mm^2)

12. Typical beam reinforcement design calculation

Direction of load application: Gravity **Load combination:** 1.2 DL + 1.6 LL

Assume,
Depth of beam = 15 in
Width of beam = 10 in
Clear cover = 1.5 in
Effective depth, d = 15-1.5 = 13.5 in

Step-1: Load calculation
Load area = 117 ft^2
Slab thickness, t = 5 in
Beam length = 15.62 ft
Compressive strength of concrete, $f'c$ = 3500 psi
Yield strength of steel, f_y = 60000 psi

Dead load:
Self-weight of slab = (5/12) * 150 = 62.5 Psf
Floor Finish, FF = 25 Psf
Partition wall, PW = 70 Psf
Dead load from slab = 62.5 + 70 + 25 = 157.5 Psf
Dead load of web portion beam = (10*10*15.62*150) / (144*1000) = 1.63 Kip

Total dead load = 157.5 * 117 + 1.63 = 18.43 Kip
Live Load, LL = 42 Psf
Total live load = 42 * 117 / 1000 = 4.914 Kip
Total ultimate load = 1.2 * 18.43 + 1.6 * 4.914 = 29.9784 Kip
Uniform distributed load, $W_{u\ UDL}$ = 29.9784 / 15.62 = 1.92 Kip/ft

Step-2: Moment calculation
Positive moment, discontinuous end is integral with the support,
+ M = $\frac{1}{14} * 1.92 * 15.62^2$ = 401.37 Kip-in
Negative moment at face of all supports (Exterior & interior)
- M = $\frac{1}{12} * 1.92 * 15.62^2$ = 468.45 Kip-in

Step-3: Effective depth check

Balanced reinforcement ratio, $\rho b = \alpha * 0.85 * \frac{f'c}{fy} * \frac{\epsilon uc}{\epsilon uc + \epsilon ut} = 0.85 * 0.85 * \frac{3500}{60000} * \frac{0.003}{0.003+.005} \beta =$ 0.015804

Maximum moment, m = $\varphi * \rho * fy * b * d^2 * (1 - 0.59 * \rho * \frac{fy}{f'c})$

$d^2 = \frac{468.45}{60 * 0.9 * 0.015804 * 10 * (1 - 0.59 * 0.015804 * \frac{60000}{3500})}$ d = 8.1 in
Which is less than effective depth.
So, adequate effective depth.

Step-4: Main Reinforcement

+ $A_S = \frac{401.37}{0.9*60*(13.5-\frac{1.15}{2})}$ = 0.57 in^2

a = $\frac{0.57*60}{0.85*3.5*10}$ = 1.15 in

- $A_S = \frac{468.45}{0.9*60*(13.5-\frac{1.36}{2})}$ = 0.68 in^2

a = $\frac{0.68*60}{0.85*3.5*10}$ = 1.36 in

$A_{s,\ min} = \frac{200}{60000} * 10 * 13.5$ = 0.45 in^2/ft

$A_{s,\ min} = \frac{3\sqrt{3500}}{60000} * 10 * 13.5$ = 0.40 in^2/ft

Choose No. 5 (No. 16) two bars at bottom (Straight).
Choose No. 5 (No. 16) three bars at extra top.

13. Typical slab reinforcement design calculation

Direction of load application: Gravity **Load combination:** 1.2 DL + 1.6 LL

Step-1: Slab Thickness

Unit weight of concrete = 150 lb/ft^3
Compressive strength of concrete, $f'c$ = 3500 psi
Yield strength of steel, f_y = 60000 psi
A = Shorter length of the slab
B = Longet length of slab
Length of clear span in short direction, l_a = 16.08 ft
Length of clear span in long direction, l_b = 23.67 ft m
= 16.08 / 23.67 = 0.679

Thickness, t = $\dfrac{\ln * (0.8+(\frac{fy}{200000})}{36+9*\beta} = \dfrac{23.67*12*(0.8+(\frac{60000}{200000}))}{36+9*\frac{23.67}{16.08}} = 6.34 \text{ in} \approx 6.5 \text{ in}$

Effective depth, d = 6.5 -1 = 5.5 in

Step-2: Load calculations

Self-weight of slab = (6.5*150)/12 = 81.25 Psf
Partition wall = 70 psf (for Floor slab)
Floor finish = 25 Psf
Live load, LL = 42 Psf
Ultimate dead load, W_{DL} = (81.25 + 25 + 70) * 1.2 = 211.5 Psf
Ultimate live load, W_{LL} = 67.2 Psf
Ultimate load, W_T = (81.25 + 25 + 70) * 1.2 + 42 * 1.6 = 278.7 Psf

Step-2: Moment calculations

Case = 4

Coefficients

$C_{a, neg}$	0.0826
$C_{b, neg}$	0.0174
$C_{A\ DL}$	0.0476
$C_{B\ DL}$	0.0101
$C_{A\ LL}$	0.059
$C_{B\ LL}$	0.0123

Positive moments:

$+M_A = C_{A\ DL} * W_{DL} * L_{A2} + C_{A\ LL} * W_{LL} * L_{A2}$
$= 0.0476 * 211.5 * 16.08^2 + 0.059 * 67.2 * 16.08^2$
$= 3.60$ k-ft/ft
$+M_B = C_{B\ DL} * W_{DL} * L_{B2} + C_{B\ LL} * W_{LL} * L_{B2}$
$= 0.010 * 211.5 * 23.67^2 + 0.0123 * 67.2 * 23.67^2$
$= 1.65$ k-ft/ft

Negative moments:

- $M_A = C_{A, neg} * W_T * L_{A2}$

  $= 0.0826 * 278.7 * 16.08^2 = 5.95$ k-ft/ft

- $M_B = C_{B, neg} * W_T * L_{B2}$

  $= 0.0174 * 278.7 * 23.67^2 = 2.72$ k-ft/ft

Step-3: Rebar for short direction/transverse direction

+ $A_{S, A} = \frac{3.60*12}{0.9*60*(5.5-\frac{0.25}{2})} = 0.15$ in^2/ft (Controlling) a = $\frac{0.15*60}{85*3.5*12}$ = 0.25 in

0.

$A_{s, min} = 0.0018 * 12 * 6.5 = 0.14$ in^2/ft

Using φ 10 mm bar

S = 0 $\frac{11*12}{0.15}$ = 8.8" ≈ 8 in (200 mm) C/C at bottom along short direction.

Crack 50% bar to negative zone.

- $A_{S, A} = \frac{5.95*12}{0.9*60*(5.5-\frac{0.42}{2})} = 0.25$ in^2/ft (Controlling) a = $\frac{0.25*60}{85*3.5*12}$ = 0.42 in

0.

$A_{s, min} = 0.0018 * 12 * 6.5 = 0.14$ in^2/ft

Already provided, $A_{s1} = \frac{0.11*12}{16} = 0.0825$ in^2/ft

Extra top required, $A_{s2} = (0.25 - 0.0825) = 0.1675$ in^2

Using φ 10 mm bar

S = $\frac{0.11*12}{0.1675}$ = 7.88" ≈ 7.5 in (190 mm) C/C extra top

Step-4: Rebar along long direction

+ $A_{S, B} = \frac{1.65*12}{0.9*60*(5.5-\frac{0.11}{2})} = 0.067$ in^2/ft

a = $\frac{0.067*60}{85*3.5*12}$ = 0.11 in

0.

$A_{s, min} = 0.0018 * 12 * 6.5 = 0.14$ in^2/ft (Controlling)

Using φ 10 mm bar

S = 0 $\frac{11*12}{0.14}$ = 9.42" ≈ 9" (225 mm) C/C at bottom along long direction.

Crack 50% bar to negative zone.

- $A_{S B} = \frac{2.72*12}{0.9*60*(5.5-\frac{0.18}{2})}$ = 0.11 in^2/ft

a = $\frac{0.11*60}{85*3.5*12}$ = 0.18 in

0.

$A_{s, min} = 0.0018 * 12 * 6.5 = 0.14$ in^2/ft (Controlling)

Already provided, $A_{s1} = \frac{0.11*12}{18} = 0.073$ in^2/ft

Extra top required, $A_{s2} = (0.14 - 0.073) = 0.067$ in^2

Using φ 10 mm bar

S = $\frac{0.11*12}{0.067}$ = 19.8" ≈ 18" (450 mm) C/C extra top.